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The Physical and Numerical Modelling of a Repaired Masonry Arch Bridge

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ABSTRACT: The behaviour of a benchmark (undamaged) and a repaired small scale arch bridge models were studied using a commercial FE package and compared to the experimental results. The bridges under consideration were $1/12^{th}$ scale models of 2-D single 6m span (in prototype) arches with span to rise ratios of 2 (deep) and 4 (shallow).

The FE model was constructed using material properties which were initially obtained from a series of tests undertaken on both the constituent and composite materials. The soil/structure interface pressures on the extrados of the arch barrel and arch deformations at different locations were predicted using the FE model under loads and compared with the experimental results. In the initial studies different masonry material failure criteria were tested and the best model selected. The bridges were repaired by applying a slab of reinforced concrete on top of the backfill. This method of repair is of interest as it has the potential to yield similar results (in strength terms) as compared to both concrete saddling and lining without the construction and environmental difficulties of either and may provide the added advantage of maintaining the benefits of masonry arching action e.g. masonry flexibility in dealing with differential settlement. Both the benchmark and the repaired arch behaviour are presented and compared with the experimental results.

1 INTRODUCTION

Masonry arch bridges are a major part of the transport system around the world. Most of the existing masonry bridges in use are more than hundred years old and therefore there is significant interest in repair and strengthening techniques. These arch bridges are now carrying much larger loads than they were ever envisaged by their original builders. In recent years experimental work has moved from assessing existing capacity towards investigations of methods to increase or maintain capacity. The effect of various strengthening methods on full scale arch was reported by Summon (Summon 1998) and Melbourne (Melbourne et al 1995). Baralos (Baralos 2002) applied different types of strengthening method to $1/12^{th}$ scale 2-D single span arch models. Some efforts were also undertaken by some researchers in using numerical methods for the analysis of these structures. The first attempt at application of the FE method to masonry arches was carried out by Towler (Towler and Sawko 1982, Towler 1985) who compared their own numerical solution with experimental results on a series of brickwork model arches. Ng et al (Ng et al. 1999) carried out FE analysis using a commercial FE package where three full scale bridge collapse tests were modelled and the results compared with available field test data. Comparisons were also made with the results obtained from other arch bridge assessment methods.

Choo et al (Choo et al. 1995) undertook an experimental and numerical investigation on a number of arch bridges at Nottingham University. Test results showed that 75mm thick layer of concrete; equivalent to a 33% increase in barrel thickness, applied to the inside of arch increased the collapse load an average by 1.9 times the pre-repaired strength. The FE study indicated that, depending on the bond condition, up to double the load carrying capacity may be expected.

A wide literature review undertaken for this study found that it was often difficult to determine the exact failure criteria used in a number of previous FE studies. It is apparent that some previous models have employed a range of different criteria and that it is not always apparent what they were.

In the current paper the results of tests on a small scale repaired arch are compared with the pre-repaired model. The FE commercial package was then used (and subsequently calibrated) to initially model the behaviour of pre-repaired arch and finally compared with the repaired experimental results.

2 MODEL DESCRIPTIONS

The model under tests were 1/12th scale replicas of a 6 metre single span three ring arch. Two types of arch geometry, with span to rise ratios of 4 and 2, were studied as typical shallow and deep arch geometries. The models were tested in a centrifuge under a steady equivalent gravity of 12g. The models were usually tested with fourteen passes of a rolling load equivalent to a scaled lorry axle and then up to the observation of first signs of failure, to enable them to be suitable for applying a repair method. The major geometrical parameters of the arches are summarised in Table 1. More details of arches under test and the test procedure can be found elsewhere (Miri and Hughes 2004).

Table 1 : Major dimensions of arch models under test

Properties	Shallow	Deep
Span(mm)	500	500
Span/rise	4	2
Ring number	3	3
Ring thickness (mm)	30	30
Load location (% span)	25	30
Depth of fill at crown (mm)	13	13
Concrete thickness (mm)	17	17

Arch deflections and soil/masonry interaction were measured during each test. The displacements were recorded using Linear Variable Direct Transducers (LVDT) and small diaphragm pressure cells were used to measure the pressure on the extrados of the arch barrel.

3 REPAIRED MODELS

Following the initial tests, the arches were repaired by laying a 17mm reinforced concrete slab on top of the backfill. The concrete itself was manufactured with 2.0mm aggregate and Ordinary Portland Cement with mix proportion of 1:1.8:2.8:0.6 (Cement: Fine: Coarse: Water) by mass.

Table 2 : Material properties of models

Properties	Bricks	Mortar	Backfill	Concrete
Compressive strength (N/mm^2)	96.0	1.7	N/A	51.0
Flexural strength (N/mm^2)	0.76	N/A	N/A	7.5
Modulus of elasticity (N/mm^2)	30000	2900	18-66	
Bulk density (kN/m^3)	22.0	18.0	20.5	21.0
Shear resistance angle	N/A	N/A	53.0	
Poisson's ratio	0.14	0.09	N/A	

Compressive strength tests on 25mm concrete cubes samples yielded an average strength of 51N/mm^2.The model concrete was nominally reinforced with a square mesh of 0.8mm diameter mild steel at 20mm centres. The material properties of models are detailed in Table 2.

4 FE MODEL

4.1 Linear FE model

The FE model of the arch bridge was constructed using the LUSAS commercial package version 13.4 (FEA Ltd 2001a) and its modeller program was used for pre and post processing of the data. 2-D plane strain elements were used to model the arch barrel and the backfill materials. Twenty two eight noded elements were used in the longitudinal direction of the arch barrel and 3 elements were used to simulate the arch barrel thickness. Different sizes of mesh were used to simulate the backfill materials. A fine mesh was used under the load position and near to the crown of the arch and a coarse mesh used far a way from the applied load location. A layer of thin interface elements was generated between the arch ring and the backfill to help account for the soil/arch interaction. The main material properties of the arch barrel, backfill and arch/fill interface which were used in the FE model are presented in Table 3.

Table 3 : The main material properties of FE model

Property	Arch barrel	Backfill	Interface	Concrete
Modulus of elasticity (N/mm^2)	4000	15	5.0	10000
Poisson's ratio	0.3	0.4	0.4	0.2
Bulk density (kN/m^3)	21.0	20.5	20.5	21.0

4.2 Non linear FE model

A non linear FE model was used to model the arch behaviour under increasing loads. The residual force norm and displacement norm were used in identifying the failure load. A modified Newton-Raphson procedure was used to deal with nonlinearities and both manual and automatic controls were used to apply the load. The self load was applied to the model using as a body force with a load factor of 12 to simulate the centrifuge acceleration and the live load was applied incrementally. By using this way of applying load the self load effect was saved as an in-situ stress and the live load effect was added to it in each increment (FEA Ltd 2001b). Different material properties and failure modes were tested to simulate the centrifuge model. Elasto-plastic behaviour was defined for the arch barrel material. The plastic behaviour of the material was simulated using a concrete crack model which is available as a pre-defined model in LUSAS.

4.3 Repaired arch FE model

Following completion of the FE benchmark model a concrete slab was added to it. Plane strain condition was also assumed for the concrete. Suitable values of modulus of elasticity, Poisson's ratio and mass density were used to identify the elastic behaviour of the slab concrete as given in Table 3. The same material model as per the barrel material cracking concrete model was used to simulate the concrete slab behaviour at failure.

5 MODEL RESULTS

5.1 Failure load results

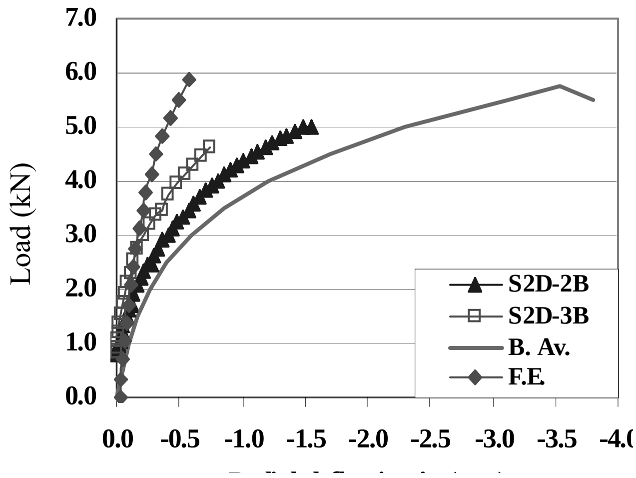
Initial results obtained from the numerical model indicated that the failure values are dependent on the convergence criteria and norm values, detailed in section 3.2. The four hinges start to form at a low load level but do not propagated fully until the non convergence of the solution stops the calculation from progressing. This suggests that full propagation of a crack along a section through the arch ring, which has previously been defined as failure criteria of the arch by some researchers (Ng and Fairfield 1999), seems not to have happened here. Therefore according to the centrifuge tests observations and initial FE model results the failure criteria was set as the load at which the forth hinge formed in the arch and that the cracks on this hinge had propagated up to 50% of the full ring thickness. The failure loads of the arch model using the above criteria with different masonry tensile strengths are given in Table 4.

Table 4 : Numerical failure load (kN) with difference tensile strength and failure criteria

Arch geometry	Shallow				Deep
Tensile strength (N/mm^2)	σ_t = 0.08	σ_t = 0.10	σ_t = 0.20	σ_t = 0.40	σ_t = 0.20
Third hinge formation	2.1	2.2	2.7	4.3	3.5
Fourth hinge formation	2.4	2.8	3.5	4.8	3.8
Forth hinge propagation to 50% ring	4.5	4.8	5.8	7.9	6.2
Forth hinge propagation to75% ring	5.5	5.8	7.2	N/A	N/A

Comparison of the numerical results obtained with the experimental centrifuge test results indicate that the closest failure load was obtained when the tensile strength of the masonry material was set to 0.20N/mm^2. Assumed strength values of about this value are typical of numerical models of masonry arch bridges (Ng and Fairfield 1999, Fanning and Boothby 2001b). Using the above failure criteria and material properties, failure loads of 5.8kN and 6.2kN are predicted for the shallow and the deep arch geometry as detailed in Table 4. In the case of the deep arch, the predicted failure load is about 10% more than the average centrifuge results for the same geometry, while there is no significant difference between the predicted and experimental values for the shallow arch.

In Fig. 1 the FE predicted load deflection for the shallow arch is compared with the experimental centrifuge results from the current study and those reported by Burroughs (Burroughs 2002) for the same geometry. Each shallow test results are referenced with the test name and Burroughs average test results are referred to B.Av. The numerical and experimental failure loads are similar (as calibrated) with the numerical model being significantly stiffer than the experimental one.



Deflection (mm)

Figure 1 : Shallow arch load deflection curve

5.2 *Pressure on arch extrados*

The calculated arch soil interface pressure values at 75% of arch span under increasing load are compared to those values from centrifuge test, for the shallow arches, in Fig. 2. Consideration of Figures shows that the predicted pressures are in range of the monitored pressures. The predicted and monitored pressures at different locations are also compared with each other for deep arch in Fig. 3.

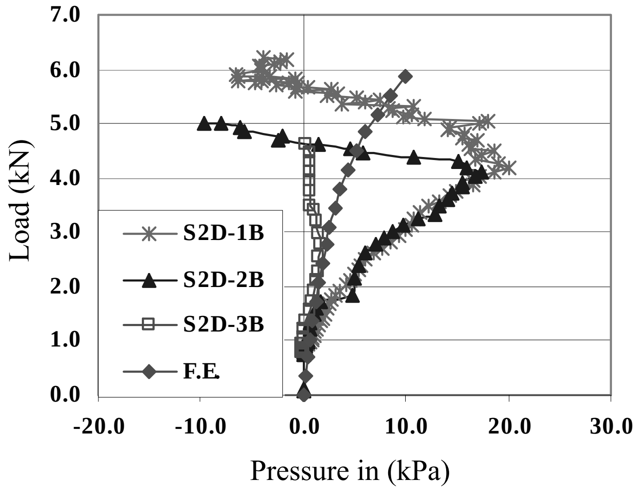


Figure 2 : Pressure on extrados of shallow arch (75 % span)

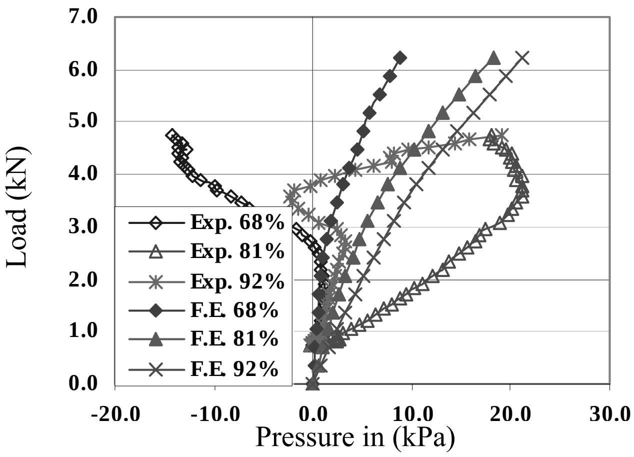


Figure 3 : Pressures at different location of deep arch

The experimental results are similar to some previous work (Burroughs, 2002) and indicate the complex nature of the way the soil moves and fails. There is evidence of elastic regions some plastic behaviour and reverse movements, thought to be associated with sudden block type movements. The FE modelling is clearly unable to capture this complex behaviour but the level of reaction provided to the arch appears appropriate.

6 PARAMETRIC STUDY

As detailed most of the material properties that were used in the FE model were obtained directly from the material test properties. The effect of material properties on the model response was also examined in FE model study. The parametric study was carried out by varying the backfill modulus of elasticity from 5000 N/mm^2 to 100000 N/mm^2 and this indicated that the resulting normal stresses only decreased from 21kPa to 17kPa on the extrados of arch barrel at 80% of arch span. Numerical studies with different plane strain and plane stress condition confirm no significant effect of this parameter. The initial studies of the model support conditions similarly indicated no significant effect of support conditions on the arch response; this might be because the boundary was sufficiently far away from the arch. The masonry tensile strength value was the most important input data in relation to its effect on the arch failure load. The effects of different tensile strength value on arch failure load are given in Table 4.

Centrifuge tests results indicated that a three fold increase in backfill depth above the crown yielded a 72% and 125% increase in the overall arch strength, for shallow and deep arches respectively (Burroughs 2002). Failure loads of 13.1kN and 15.5kN were predicted by the FE model for shallow and deep arch which indicates an increase of 130% and 167% for this increase in backfill depth. For both geometries, the predicted failure loads are more than that measured in the experiments.

7 REPAIRED MODEL RESULTS

The repaired arch model conditions at failure and opening crack positions for the shallow arch geometry are presented in Fig. 4. The experimental cracks positions are also included.

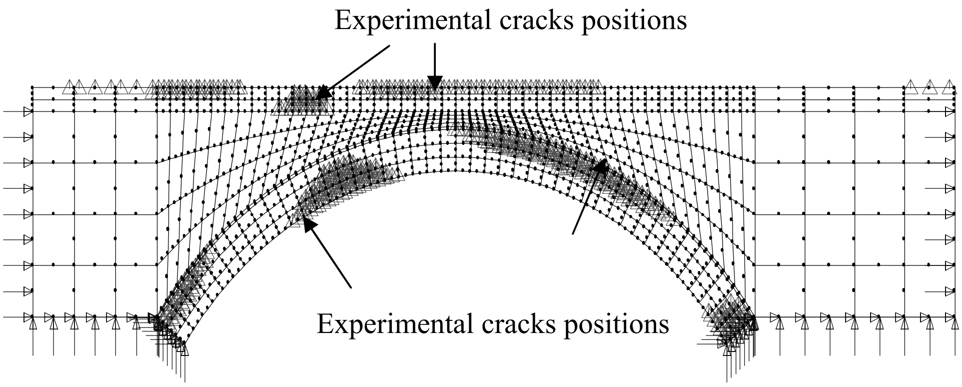


Figure 4 : Shallow repaired arch predicted hinge position

Comparison between the experimental and numerical results indicates that the numerical model simulates the general overall behaviour of the arch model very well and predicted cracks positions are consistent with the experimental results. Failure loads of 15.2kN and 14.6kN for shallow and deep repaired arch geometries were obtained using the FE model which is approximately the same as the experimental results for shallow arch, but is about 12% less than the experimental for deep geometry. Load arch deflection curve for both shallow and the deep repaired arches are presented in Figs. 5 and 6.

Consideration on theses figures indicates significantly higher deflections in the experimental results. However the FE model simulates the arches behaviour with a reasonable accuracy up to 50% of the failure load but at the higher loads the experimentally obtained deflections are significantly more than the FE models. This shows that the FE model did not predict the arch response well under high load level particularly as the failure load is approached.

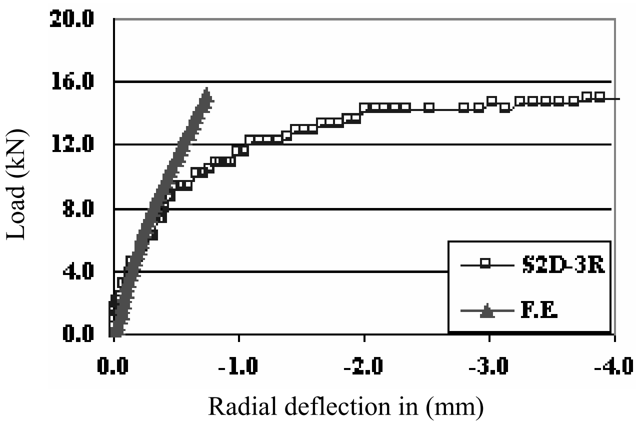


Figure 5 : Load arch deflection (Repaired shallow arch)

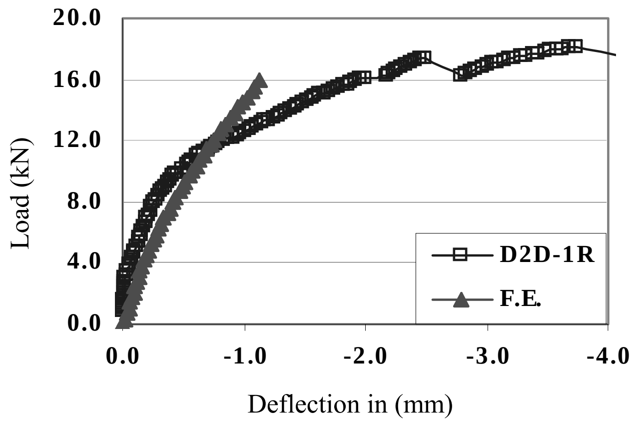


Figure 6 : Load arch deflection (repaired shallow arch)

In Fig. 7 the repaired arch barrel extrados pressures at different locations are compared with the numerical results for the same sections under increasing loads. In both the sections there is reasonable consistency between two methods.

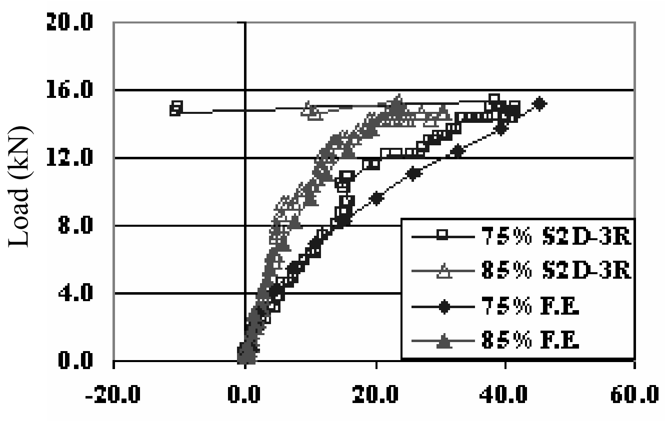


Figure 7 : Pressures at different sections of shallow repaired arch

8 CONCLUSIONS

The following conclusions drown from the numerical and experimental studies:

- The 2-D FE model can simulate the general behaviour of arch particularly in the prediction of the hinge development.
- There are issues associated with failure criteria for FE models in the literature review undertaken for this study. It can be difficult to replicate previous studies where full data on the FE studies are not detailed especially in relation to the failure criteria.
- In terms of the predicted value of the failure load, the FE model predicted the arch failure loads for both geometries well but in terms of deflections the FE model is significantly stiffer than the experimental models.
- The pressures on the extrados of arch barrel determined using the FE model are in good agreement with those obtained from the experimental models however some difference were observed particularly near to the failure loads.
- The use of a surface slab produces a similar strength enhancement as a slab bonded to the arch barrel but via a different support mechanism.

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